Title: Innovative Design for the Colton Flyover Grade Separation of UPRR and BNSF, Colton, CA

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ABSTRACT

The Colton Crossing Design Team developed an innovative approach for grade separating the Union Pacific-BNSF quadruple diamond at-grade crossing at Colton, California. The site lies in a developed area bounded by Interstate 10 on the north and residential/commercial neighborhoods on the south. Due to the need to maintain operations on existing tracks during construction and close proximity to I-10, traditional soil embankment fill with side slopes was not feasible for most of the project limits. Since the grade separation required retaining walls as high as 40 feet to elevate UP tracks over BNSF, high bearing pressures and expensive retaining walls were expected. The retaining walls and fill were estimated to be about 40% of the total cost of the project. In addition, Colton lies within an active seismic zone and the nearby San Jacinto fault has the capability of producing very large ground motions in an earthquake. The weight of concrete retaining walls and soil fill would result in large inertial forces, high peak bearing stresses and the potential for catastrophic failure of the walls and foundation soils in a seismic event.

A novel approach to the retaining walls and fill was developed by the Design Team which proposed the use of lightweight cellular concrete fill with precast concrete panel fascia walls to form a cellular concrete retaining structure. The Design Team developed design criteria since no AREMA or AASHTO guidance was available for this type of structure. The Design Team performed advanced service and seismic design utilizing finite element analyses as well as other
traditional methods. The analyses verified that the expected performance and construction costs were reduced considerably. This innovative cellular concrete retaining structure has the following advantages:

- Lighter weight and stronger than compacted soil backfill.
- Built using small equipment and without importing a large volume of fill, thereby reducing impacts to the environment.
- No lateral earth pressures and very small live load surcharge pressures on bridge abutments and side facing walls.
- Reduced seismic inertial effects due to the reduction in mass, as well as enhanced seismic stability due to the large block-like behavior of the retaining structure.
- Greatly reduced anticipated settlements, especially in combination with shallow ground improvement.
- Cost savings of up to 30% compared to conventional cast-in-place concrete walls with tie rods and soil fill.

OVERVIEW

Union Pacific Railroad (UP) has a shared at-grade crossing with BNSF Railway in the City of Colton, San Bernardino County, California. The crossing of these tracks is referred to as the “Colton Crossing”. The Colton Crossing was established in 1882, and is used by BNSF Railway (BNSF), north/south tracks, and Union Pacific Railroad (UPRR), east/west tracks, for goods movement, in addition to Metrolink and Amtrak for passenger service. More than 110 trains passed through Colton Crossing daily in 2008 – making it one of the busiest at-grade rail-to-rail crossings in the nation (coltoncrossing.com).
This project provides grade separation for these two heavy traffic lines and increases the efficiency of traffic flow in this area. This grade separation, or “flyover”, structure consists of a cellular concrete embankment and a series of bridges along the alignment of the UP tracks. The total length of the Colton Crossing Flyover Retaining Structure is about 7,000 feet with an embankment width of about 50 feet and a maximum height of about 40 feet. See Figure 4 below.

CONSTRAINTS

Operational, physical and phasing constraints governed the construction of the project. These constraints dictated the limits of the possible embankment footprint.

Operational Constraints. The UP and BNSF mainlines and connection tracks experience heavy traffic. The heavy traffic prevented construction of a grade separation structure on the existing alignment due to interference with the UP mainline. The tie-ins between the new project and the existing alignment had to be designed to minimize delays to traffic during tie-ins. In
order to minimize delays to construction, the project was designed to minimize work in the 15-foot safety zone. In this zone, construction work has to stop when trains are passing.

**Physical Constraints.** The physical constraints for the grade separation construction consisted of Interstate 10 to the north, neighborhoods on the southwest and an existing rail yard to the southeast. These constraints limited where the new alignment could be placed. The UP owned additional right-of-way north of its existing tracks (north of existing Mainline No. 1). The north right-of-way was constrained from the BNSF crossing to the west at Rancho Avenue by the I-10 right-of-way.

The project was further constrained by the Rancho Avenue overhead grade separation to the west and the Mount Vernon Avenue overhead grade separation to the east. These roadways over railroad grade separations control the location of the new alignment tie-ins both vertically and horizontally.

![Figure 2. Typical T-Wall Mainline Section Relative to Existing Main Track.](image)
Phasing Constraints. The phasing of the construction was constrained by the track laying machine availability. Each track could have been shifted to the new alignment in two different construction phases. But due to the availability of the track laying machine, both new mainline tracks would have to be constructed on the new alignment at the same time.

These constraints dictated that the new alignment be placed to the north of the existing alignment. Additional right-of-way was purchased from Caltrans between Rancho Avenue and the BNSF Mainlines for this project.

These constraints would not allow the use of typical soil embankment side slopes for the flyover construction. Therefore, vertical sides were required for the embankment construction for the entire south side and the north side from Rancho Avenue to La Cadena Avenue.

Alternative Discussions. During the 25% design phase of the project, several alternatives were considered for the construction of the embankment. These included: conventional cast-in-place concrete retaining walls with soil backfill, lightweight backfill, mechanically stabilized earth walls and gravity wall systems.
The conventional cast-in-place concrete retaining walls utilizing soil backfill would require a tie rod system to resist the soil loads, including seismic loads, and live load surcharge. The upper 15 feet of loose soil in the existing site subsurface profile would need to be removed and replaced with an engineered fill to accommodate the high required bearing pressures and tight tolerances on settlement. This foundation system was used as the base design. All other systems were compared to this ‘base’ design for cost and constructability.

Lightweight backfills were also considered for use. These included geofoam units and cellular concrete. The geofoam was eliminated from further consideration due its potential reactivity to hydrocarbons and high relative cost. Cellular concrete was considered as a viable alternative for lightweight backfills.

Mechanically stabilized earth walls were not considered due to the difficulty and long duration of repairs if damaged. Panel replacement at lower levels would require that one or both mainlines be out of service for an extended time. The panel systems associated with the walls was considered further with use of cellular concrete fill instead of traditional granular soil backfill.

Gravity wall systems such as bin walls and crib walls could not be used as their front faces are generally battered. This batter would increase the embankment footprint impacting the project constraints. Patented systems such as the T-Wall have not been used to the embankment heights required.

Elements from the various systems considered were combined in the final design. The cellular concrete was selected using the panels from a mechanically stabilized earth wall system. The cellular concrete had a minimum height constraint of 8 feet due to desired cellular concrete placement depths. Also, parts of the south wall on each end had to be constructed in phases.
Therefore, a precast concrete gravity wall system was selected for walls with heights below the minimum height of 10 feet.

**Embankment section.** The flyover embankment structure supports two railroad tracks and a maintenance access road, which was planned with lateral clearance for a possible future third track. The embankment section consists of, in descending order: 8.5-foot wide concrete ties with ballasted track section [12 inches ballast/18 inches subballast], 3-foot thick upper layer of Class IV cellular concrete, variable thickness of Class II cellular concrete, 2.5-foot thick Class IV layer of cellular concrete with a 4-foot deep shear key embedded in the foundation soils (at higher embankment sections), and vibro-replacement stone columns approximately 15 ft deep in the foundation soils.

Cellular concrete was selected for the embankment based upon its low density and relatively high compressive and shear strength when compared to earthen fill. Use of lightweight material reduces the bearing pressure and the static and dynamic lateral earth pressures in the bridge abutment areas. A typical section of the proposed embankment structure with ground improvements is shown in Figure 4. Cellular concrete can also be pumped long distances for...
placement within a small footprint by the on-site equipment, which is critical in a tight construction site between Interstate-10 and active railroad tracks.

**Cellular concrete.** Cellular concrete is an engineered, low density material having a homogeneous cell structure formed by the addition of prepared foam or by the generation of gas within the fresh cementitious mixture. It is usually cast in densities ranging from about 20 to 120 pounds per cubic foot (pcf). The air cells created by the preformed foam may account for up to 80% of the total volume (Fouad, 2006).

Based upon research by Sandia National Laboratories (Sandia, 2004), the tensile to compression strength ratio is approximately 10% and the stress-strain relationship of unreinforced cellular concrete is similar to that of conventional concrete. Also, based upon available technical literature (Cellular Concrete LLC, 2011), the wet cast densities for Class II and Class IV cellular concrete are about 30 and 42 pounds per cubic foot (pcf), respectively; and the dry densities are about 25 and 37 pcf, respectively. The corresponding as-cast compressive strengths for these densities are about 40 and 120 pounds per square inch (psi), respectively; and

![Figure 5. Close up Cellular Concrete Structure. Source: ACI 523.1 R-06 “Guide for Cast-in-Place Low Density Cellular Concrete”](image)

![Figure 6. Typical Placement of Cellular Concrete. Source: ACI 523.1 R-06 “Guide for Cast-in-Place Low Density Cellular Concrete”](image)
the 28-day compressive strengths are about 140 and 412 psi, respectively.

**Ground Improvement and Shear Key.** Due to the variability of the stiffness of the upper foundation soils supporting the new embankment, it was recognized that ground improvement would be needed to provide a uniformly stiff pad and to minimize the potential for differential settlements from earthquake shaking. Vibro-replacement stone columns were selected as the ground improvement method to increase the relative density and shear strength of the subgrade and shallow foundation soils, and to improve the overall stability of the embankment structure.

The vibro-replacement technique utilizes a small mobile rig to insert a vibrating probe for the construction of stone columns of depths up to 30 feet below the ground surface. In addition to the ground improvement, a shallow shear key consisting of light-cellular concrete was bedded in the foundation soils to improve the sliding resistance of the embankment during an earthquake (Figure 4). The influence of these improvements on the overall stability of the embankment structure will be discussed subsequently.

The ground improvement resulted in an increase in stiffness of the matrix soil, such that the composite internal friction angle of the improved ground was increased. This was verified in the

**Figure 7. Installation of Vibro-Stone Columns at the Site.**
field with SPT Testing (ASTM D1586) at regular intervals across the alignment throughout the full depth of the improved soil block.

SITE CHARACTERIZATION

Existing subsurface conditions. Geotechnical investigations for this site were performed by C.H.J. Inc. (C.H.J.) during the summer of 2010. The conditions at this site consist of [top to bottom]: thin layer of loose, silty sandy fill [1 to 9 feet thick], loose to medium dense silty sand [5 to 25 feet thick], medium dense silty sand [0 to 35 feet thick], and dense to very dense silty sand [thickness unknown, bottom of boreholes] (C.H.J., 2011). Groundwater was located between 117 and 123 feet below the ground surface as measured in the boreholes.

Figure 8 presents a subsurface profile along the centerline of the proposed flyovers structure near the location of the highest embankment fill.

Figure 8. Subsurface Profile near Maximum Height Embankment Section.
Shear wave velocity measurements were attempted at the site via the seismic cone penetrometer (SCPT). However, such measurements were not presented in the geotechnical report because of the excessive background noise caused by traffic on the adjacent freeway. The probabilistic seismic hazard analysis (PHSA) assumed that the site classified as Site Class D (stiff soil profile) with an estimated average shear wave velocity of 270 m/s in the upper 100 feet (C.H.J., 2011).

Seismic Hazard. The site is not located in a mapped fault rupture zone (C.H.J., 2011); however due to the presence of several nearby, active faults the expected ground motion at this site is large. The project geotechnical report provided acceleration response spectra for the three probabilistic design basis earthquakes (Figure 9). The Level 1, 2 and 3 events represent spectral accelerations having average return periods of 72, 475 and 2475 years, respectively. [Note: the calculated vertical design accelerations at this site are considerably higher than would be encountered in most other areas of California. For example, vertical Peak Ground Acceleration (PGA_v) is on the order of 90% of the corresponding horizontal value based on published attenuation relationships (Campbell-Bozorgnia, 2008)].

The controlling fault at this site for the Level 3 event is the San Jacinto fault zone (SJFZ) based on the seismic deaggregations of PGA and the 0.2 sec spectral acceleration values. Depending on the fault rupture scenario, the controlling earthquake is approximately M7.0 to M7.7 event. The San Jacinto fault is the closest known active fault to the site and is

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about 1.4 km northeast of the planned alignment (C.H.J., 2011). C.H.J. (2011) reports that the SJFZ is a system of northwest-trending, right-lateral, strike-slip faults that traverses the southwestern San Bernardino Valley and indicates a zone of active surface faults, tectonic deformation and folding.

**DESIGN AND PERFORMANCE CRITERIA**

Performance criteria established for the Flyover Retaining Structure adopted recommendations from AREMA (2010) for bridges, which include three levels of ground motion with corresponding performance goals as follows: (1) Level 1 – The embankment structure should remain intact with no permanent deformation (i.e. the seismic loads must remain within the elastic range of the stress-strain curve of the embankment); (2) Level 2 – The embankment structure should be repairable, with only minor permanent deformation; and (3) Level 3 – The embankment structure must not collapse after experiencing permanent deformations.

**PRELIMINARY STABILITY ANALYSES**

Comprehensive seismic design guidance for free standing lightweight concrete embankments has not been fully developed in the U.S, but a rational numerical modeling approach applicable to the seismic design and evaluation of freestanding geofoam embankments is discussed in Bartlett and Lawton (2008) and Bartlett et al. (2011). Principals from this approach were applied in the preliminary analyses.

Preliminary global stability analyses of potential deep-seated failure surfaces were performed using the computer program SLOPE/W (GEO-SLOPE, 2007). The SLOPE/W program uses limit equilibrium techniques to search for the critical failure surface (i.e., that surface with the minimum factor of safety). The inertial acceleration used in the analyses for the embankment
corresponded to the spectral horizontal acceleration value computed at the fundamental period of the embankment, $T_0$. The corresponding inertial acceleration was not varied in the embankment because the light-weight cellular concrete embankment behaves more like a rigid body under elastic conditions. Various values of $T_0$ were calculated along the alignment according to the methodologies discussed in Horvath (2004), Bartlett and Lawton (2008), and Bartlett et al. (2011) using the corresponding height, width, and elastic properties of the embankment at that location.

Ultimately, it was found that the highest embankment cross-section of about 40 feet controlled the design because this maximum height produced the highest inertial forces within the embankment and the lowest factors of safety against basal sliding. The corresponding $T_0$ value for the controlling section is about 1.15 seconds. The corresponding design horizontal spectral accelerations for each AREMA level earthquake are shown in Figure 10. These values were used as the horizontal inertial acceleration in the limit equilibrium analyses.

For cases where the computed global stability factor of safety (FS) was below 1.0, a Newmark sliding block displacement analyses (Kramer, 1996) was also performed using SLOPE/W to provide preliminary estimates of the permanent deformation for each of the AREMA earthquake events. Using the computed accelerations at $T_0$ for the AREMA Level 1 event (Figure 8), the
computed minimum FS was 2.0. This high factor of safety suggests no yielding will take place within the embankment structure or the foundation soils. Similarly, for the AREMA Level 2 and 3 earthquake events, the computed FS were 0.7 and 0.4, respectively, which suggest yielding, predominately in the foundation soils. The resulting permanent displacements were estimated as 1 and 7 inches for the AREMA Level 2 and 3 events, respectively.

In addition to these simplified limit equilibrium analyses, the global seismic stability of the embankment structure was also investigated using the finite element method and computed stress time history calculated from QUAKE/W (GEO-SLOPE, 2007) for the controlling earthquake record. These latter results were found to agree very closely with the limit equilibrium global stability analyses using the spectral accelerations from Figure 10.

DEVELOPMENT OF STRONG GROUND MOTION

More elaborate deformation analyses were also conducted that required horizontal and vertical acceleration time histories for the design basis events. These were developed as spectrum compatible time histories using spectral matching techniques from the computer program RSPMATCH (Abrahamson, 1998). The goal of spectral matching is to generate a set of realistic time histories that satisfy seismological and geological conditions appropriate for the seismic source and site conditions at the candidate site. The main considerations for selecting time histories are: (1) appropriate earthquake magnitude, (2) faulting mechanism (e.g., strike-slip, vs. dip-slip, etc.), (3) tectonic regime, (4) source-to-site distance, and (5) geological structure. The candidate records should be selected from earthquake events that have similar conditions, whenever possible.

Selection of Time Histories. Candidate strong motion records were selected from the Pacific Earthquake Engineering Research (PEER) Center Strong Motion website based on earthquake
magnitude, source distance and faulting style that was similar to the candidate site. In addition to
the spectral shape and amplitude, selection of time histories should consider the dependence of
the system response on time domain characteristics such as earthquake duration, pulse shape,
pulse sequencing, etc. AREMA (2010) does not specify the number of time histories required
for site-specific, time-domain, nonlinear analysis. However, it is recommended that at least 7
independent time histories be used for such analyses.

In addition, we recommend the candidate acceleration time histories should be obtained from
rock or very stiff soil sites (Site Class B or C), whenever possible, and should be statistically
independent motions (i.e., should have no statistical or spatial correlation). Synthetically
generated time histories are not recommended for ground response analyses because such
records may not have near field and other site effects, which may be important for non-linear
time domain analyses of nearby, large earthquakes.

The time histories selected for our analyses were not modified for duration. This was deemed
unnecessary, because the selected time histories have approximately the same earthquake
magnitude and distance from the seismic source distance as the controlling earthquake at the
Colton Crossing site.

Spectral Matching. Most acceleration time histories, when taken at face value without
modification, do not provide an adequate match to the design spectrum, thus they must be scaled,
adjusted, or matched to provide a better fit. Spectral matching may be done in either the time
domain or the frequency domain in such a way that the spectral acceleration values of the
spectrally matched time history match a target response spectrum within a prescribed tolerance.
The spectral matching performed by RSPMATCH for this project was done throughout the full
spectral range with 5 percent error tolerance. For example, Figure 11 shows the horizontal
response spectral results from RSPMATCH, and the corresponding spectrally-matched time histories are shown in Figure 12. Vertical time histories used in the load-deformation analyses discussed below followed the same process.

**Figure 11. Spectrally matched time histories for Level 3 horizontal response**

**Figure 12. Comparison of spectrally-matched time histories with Level 3 target spectrum.**

**Baseline Correction.** A baseline correction should be performed on the input time histories after spectral matching. The spectral matching process may introduce some drift into the
processed record, which must be corrected. Seismosignal (http://www.seismosoft.com/en/SeismoSignal.aspx) was used to baseline correct the spectrally-matched time histories.

**Deconvolution.** The spectrally-matched, strong motion records were deconvolved to a depth equal to the base of the 2D numerical model using the 1D equivalent linear procedures described by Mejia and Dawson (2006). The steps and boundary conditions required to convolve the motion upward through the 2D model are described later. The deconvolution analysis was done with PROSHAKE™ using linear-elastic soil properties for the foundation soils. (Preliminary analyses showed that if nonlinear soil properties were assigned to the deconvolution model, then the deconvolution analysis produced numerical instability due to the large amplitude of the input motion.) Thus, the foundation soils in the model were treated as linearly elastic materials in both the deconvolution and subsequent convolution analyses. The deconvolution motion was convolved back to the surface to verify that the deconvolution-convolution analyses were capable of reproducing the original spectrally-matched time history at the ground surface. This step ensured that the design basis ground motion was successfully delivered to the base of the embankment without any amplification or attenuation of the spectral values. However, the drawback to this procedure is that potential inelastic deformation of the foundation materials could not be estimated.

**FINAL SEISMIC STABILITY AND LOAD-DEFORMATION ANALYSES**

The computer programs QUAKE/W (Geostudio, 2007) and FLAC 2D (Itasca, 2005) were used to perform the seismic stability and load-deformation analyses of the embankment. The finite element option in QUAKE/W was used to model the embankment and foundation soil elastically, with no basal sliding interface, thus maximizing the seismic stresses induced in the
embankment structure. The computer program FLAC was used to analyze the potential for yielding within the embankment, basal sliding, uplift and rocking of the embankment and to model the performance of the proposed ground improvement in the foundation soils.

**QUAKE/W Equivalent Linear Elastic Analyses.** In QUAKE/W, the Direct Integration Method is used to compute the motion and predict excess pore-water pressures, if groundwater were present, resulting from inertial forces at user-defined time steps. Tables 1a and 1b present the maximum computed accelerations and stresses at various “history” points of interest (points A through G), for all of the design strong ground motions described previously. The history points were taken at the following locations within the embankment structure (see Figure 13):

- **Point A** - “Quiet” point located at the ground surface far away from the embankment (used to check the computed ground acceleration and verify that the record has been deconvolved accurately);
- **Point B** – Top of embankment at centerline (compare this to computed fundamental period of the embankment);
- **Point C** – Center of Class II cellular concrete layer at the centerline of the embankment;
- **Point D** – Class II and Class IV cellular concrete interface at the centerline of embankment;
- **Point E** – Bottom of class IV cellular concrete shear key at the centerline of embankment;
- **Point F** – Precast panel/ footing interface; and

![Figure 13. History Point Definition](image-url)
• Point G – Bottom of panel footing at soil interface

Table 1a – Summary of Seismic Accelerations and Stresses at Level 2 Event

<table>
<thead>
<tr>
<th>History Point</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
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<tbody>
<tr>
<td>X-Acceleration (%g)</td>
<td>0.85</td>
<td>0.85</td>
<td>0.72</td>
<td>0.66</td>
<td>0.7</td>
<td>0.68</td>
<td>0.67</td>
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<tr>
<td>Y-Acceleration (%g)</td>
<td>0.96</td>
<td>0.32</td>
<td>0.30</td>
<td>0.28</td>
<td>0.28</td>
<td>0.56</td>
<td>0.55</td>
</tr>
<tr>
<td>XY-Shear Stress (psf)</td>
<td>245</td>
<td>159</td>
<td>948</td>
<td>1,198</td>
<td>1,147</td>
<td>2,995</td>
<td>4,817</td>
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<tr>
<td>X-Stress (psf)</td>
<td>498</td>
<td>243</td>
<td>162</td>
<td>492</td>
<td>790</td>
<td>5,991</td>
<td>6,785</td>
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<tr>
<td>Y-Stress (psf)</td>
<td>572</td>
<td>230</td>
<td>1,198</td>
<td>1,790</td>
<td>2,200</td>
<td>2,749</td>
<td>8,597</td>
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Table 1b – Summary of Seismic Accelerations and Stresses at Level 3 Event

<table>
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<th>History Point</th>
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<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
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<tr>
<td>X-Acceleration (%g)</td>
<td>1.72</td>
<td>1.48</td>
<td>1.19</td>
<td>0.98</td>
<td>0.97</td>
<td>0.97</td>
<td>0.97</td>
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<td>Y-Acceleration (%g)</td>
<td>1.50</td>
<td>0.50</td>
<td>0.47</td>
<td>0.44</td>
<td>0.44</td>
<td>0.92</td>
<td>0.90</td>
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<tr>
<td>XY-Shear Stress (psf)</td>
<td>495</td>
<td>273</td>
<td>1,607</td>
<td>1,984</td>
<td>1,789</td>
<td>4,590</td>
<td>7,315</td>
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<tr>
<td>X-Stress (psf)</td>
<td>693</td>
<td>275</td>
<td>196</td>
<td>816</td>
<td>1,113</td>
<td>9,296</td>
<td>10,568</td>
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<tr>
<td>Y-Stress (psf)</td>
<td>712</td>
<td>263</td>
<td>1,382</td>
<td>2,058</td>
<td>2,518</td>
<td>3,460</td>
<td>12,404</td>
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</table>

According to AREMA (2010), the maximum useable strain at the extreme concrete compression fiber is equal to 0.003 in/in at concrete crushing, which corresponds to the ultimate load or extreme event. The maximum shear strength of unreinforced cellular concrete was assumed to be, \( V_c = 0.75x2x\sqrt{f'c} \) per ACI 213R-87 (ACI, 1999). Based upon these criteria, the maximum shear stress for the Class II and Class IV cellular concrete are 2,556 and 4,384 psf,
respectively. Therefore, for the estimated stresses at the history points listed in Tables 1a and 1b, the embankment should remain elastic during and following the AREMA Level 2 and Level 3 earthquakes. It should be noted that history points F and G were located near the concrete panel/footing interface and the maximum shear stress of 18,215 psf was assumed based upon 4,000 psi compressive strength concrete for this material.

The ultimate compressive stresses (Y-Stress) of Class II and Class IV cellular concrete are 20,160 psf (140 psi) and 59,328 psf (412 psi), respectively as discussed previously. Neither the AREMA Level 2 or Level 3 earthquake analyses produced compressive stresses in excess of the cellular concrete’s allowable compressive strength, which was taken as 30% of the ultimate compressive strength ($0.3f'_c$) based on AREMA Section 2.26.1 for bearing on a loading area.

**FLAC Non-Linear Analyses.** The finite difference method is employed by FLAC (Itasca, 2005) which allowed the investigation of other potential failure mechanisms beyond those analyzed in QUAKE/W. The primary advantage that FLAC offered was its interface nodes, which allowed for sliding and separation between dissimilar materials. FLAC was used to assess: 1) internal shear and tensile failure within the embankment; 2) basal sliding at the embankment/foundation interface; 3) excessive rocking/overturning; and 4) dynamic bearing capacity failure of the improved foundation for the AREMA Level 2 and 3 earthquake events.

The results of the FLAC analyses indicate that the cellular concrete embankment and shear key remain within the elastic range in compression, tension and shear for all Level 2 and 3 earthquake time histories. In addition, the FLAC analyses suggest that only a minor amount of permanent basal sliding is expected (i.e., 4 and 6 inches maximum for the Level 2 and 3 earthquakes, respectively). These same analyses also indicate that rocking and uplift are not important failure modes, because no significant uplift is occurring at the basal corners and these
areas are not being damaged (i.e., not yielding). However, the FLAC analyses show that the underlying ground improvement does not remain in the elastic range, but will experience some yielding, which produces about 6 inches of horizontal displacement at the embankment/treated soil interface for the Level 3 earthquake.

**Comparison of QUAKE/W and FLAC models.** The results of the QUAKE/W and FLAC models were found to be very complimentary and similar to each other. Both models were able to convolve the design ground motions to the surface accurately. In addition, the results of both models indicate some deformation of the shear key at its interface with the soil. Also, as previously discussed, the QUAKE/W model was found to be very useful in estimating conservative seismic accelerations and internal forces at various points throughout the embankment and soil structure. The FLAC model was very useful in evaluating the permanent deformation of the embankment and its interface with the foundation.

**FINDINGS**

With the incorporation of the proposed ground improvement and light weight cellular concrete shear key, seismic global stability analyses indicate that the cellular concrete embankment will remain stable under AREMA Level 1 seismic loading (F.S. > 1.0), and the estimated permanent displacement of the highest embankment structure section is expected to range from 1 to 4 inches at the Level 2 earthquake, and from 4 to 7 inches at the Level 3 earthquake.

Seismic load-deformation analyses indicate that the cellular concrete embankment will not yield under any of the AREMA Level 1, 2 and 3 earthquakes, and its response should remain elastic. Also, these analyses also indicate that the shear key is integral to limiting the basal sliding of the embankment structure and is an important design feature. Additionally, the vibro-
replacement columns also help in limiting the sliding and deformation of the foundation soil in a secondary role compared to the benefit gained by the shear key.

CONCLUSION

This project was constrained by a number of operational, physical, and phasing constraints that factored into the design and construction of the Colton flyover project. Furthermore, demanding seismic design accelerations and the associated inertial forces created a unique and multi-faceted problem requiring an equally unique and challenging solution. Ultimately, a seismic design procedure was developed and implemented for this project based upon past experiences with similar geomaterials and applications.

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Colton, CA

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Outline of Presentation

- Project Overview
- Project Constraints
- Retaining Structure Design
- Established in 1882, Colton Crossing is located in the City of Colton just south of Interstate 10 about a quarter mile east of Rancho Avenue.

- Used by BNSF Railway (BNSF), north/south tracks, and Union Pacific Railroad (UPRR), east/west tracks, for goods movement, in addition to Metrolink and Amtrak for passenger service.

- More than 110 trains passed through Colton Crossing daily in 2008 – making it one of the busiest at-grade rail-to-rail crossings in the nation.
• **Needs for Railroad Crossing Improvement:**
  – Continuous movement of goods to and from Port of Los Angeles and Long Beach inhibited by heavy railway and roadway traffic.
  – Delays, reduced efficiency and reliability of in passenger transit.
  – Sustained economic growth to region and beyond

• **Proposed Project:**
  – Eliminate at grade crossing with 7,000-foot long flyover consisting of embankment and bridge structures.
Outline of Presentation

- Project Overview
- Project Constraints
- Retaining Structure Design
Constraints

• Operational
• Physical
• Phasing
Constraints

- Operational
  - Maintain Two Mainlines
  - Minimize Track Closures for tie-ins
  - Minimize Work within 15 feet of Tracks
• Physical
  – Right-of-Way
  – I-10
  – Rancho Avenue and Mount Vernon Avenue
• Phasing Constraints
  – Track Laying Machine Availability – Place both new tracks at the same time
Constraints Resolution

2012 Annual Conference & Exposition

September 16-19, 2012 ● Chicago, IL
Constraints Resolution

• Solution:
  – 40-ft high by 50-ft wide freestanding railroad embankment with vertical sidewalls
  – Vibro-Replacement Stone Column foundation improvement
  – Shear key
  – Embankment comprised of lightweight cellular concrete
Constraints Resolution
Outline of Presentation

- Project Overview
- Project Constraints
- Retaining Structure Design
Retaining Structure Design

1. Site Characterization
2. Design Criteria
3. Preliminary Stability Analyses
4. Development of Strong Ground Motion
5. Final Seismic Stability and Load Deformation Analyses
6. Findings and Conclusions
Vibro-Replacement Stone Columns
What is Cellular Concrete?

- Low density material
- Homogeneous cell structure formed by the addition of prepared foam or by the generation of gas within the fresh cementitious mixture.
- Usually cast in densities ranging from about 20 to 120 pounds per cubic foot (pcf). The air cells created by the preformed foam may account for up to 80% of the total volume (Fouad, 2006)

Source: ACI 523.1 R-06 “Guide for Cast-in-Place Low Density Cellular Concrete”
2.2 Properties

2.2.1 The LDCCF shall meet the following properties:

<table>
<thead>
<tr>
<th>CATEGORY</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Cast Density, pcf</td>
<td>30</td>
<td>36</td>
<td>42</td>
</tr>
<tr>
<td>Minimum Compressive Strength, psi</td>
<td>40</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>Freeze-Thaw Resistance, Cycles</td>
<td>330</td>
<td>-</td>
<td>330</td>
</tr>
<tr>
<td>Relative E not less than 70%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>per ASTM C666, modified</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Modulus, G. psi</td>
<td>27,670</td>
<td>41,800</td>
<td>-</td>
</tr>
<tr>
<td>per ASTM D4015 at</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>confining stress of 3 psi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young's Modulus, E, psi</td>
<td>67,500</td>
<td>101,900</td>
<td>-</td>
</tr>
<tr>
<td>based on Poisson's Ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \nu=0.22 ) and ( E=2G(1+\nu) )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Strength / Density Chart

These material weights and measures are for one individual cubic yard of cellular concrete. Multiply these amounts by the number of cubic yards you wish to batch for your project.

### Typical Neat Cement (No Sand) Mixes

The following chart illustrates the various typical properties of Weight Density (lb./c.f.), Compressive Strength (psi), and Thermal Conductivity values attainable with various volumes of preformed foam additions to Neat Cement Mixes.

<table>
<thead>
<tr>
<th>Wet Cast Density (lb/ft³)</th>
<th>Dry Density (lb/ft³)</th>
<th>Compressive Strength (28 Days) (psi)</th>
<th>&quot;k&quot; Thermal Conductivity (Btu in/h ft² °F)</th>
<th>Portland Cement (lbs/yd³)</th>
<th>Foam Volume (ft³/yd³)</th>
<th>Foam Liquid Concentrate Weight (lb/yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>16</td>
<td>50</td>
<td>0.54</td>
<td>328</td>
<td>22.7</td>
<td>2.17</td>
</tr>
<tr>
<td>25</td>
<td>20</td>
<td>80</td>
<td>0.60</td>
<td>420</td>
<td>21.5</td>
<td>2.06</td>
</tr>
<tr>
<td>30</td>
<td>25</td>
<td>140</td>
<td>0.67</td>
<td>512</td>
<td>20.3</td>
<td>1.94</td>
</tr>
<tr>
<td>35</td>
<td>29</td>
<td>210</td>
<td>0.76</td>
<td>603</td>
<td>19.1</td>
<td>1.83</td>
</tr>
<tr>
<td>40</td>
<td>34</td>
<td>330</td>
<td>0.87</td>
<td>695</td>
<td>17.9</td>
<td>1.71</td>
</tr>
<tr>
<td>45</td>
<td>38</td>
<td>450</td>
<td>0.98</td>
<td>787</td>
<td>16.7</td>
<td>1.60</td>
</tr>
<tr>
<td>50</td>
<td>43</td>
<td>640</td>
<td>1.06</td>
<td>878</td>
<td>15.5</td>
<td>1.48</td>
</tr>
<tr>
<td>55</td>
<td>47</td>
<td>790</td>
<td>1.20</td>
<td>970</td>
<td>14.3</td>
<td>1.37</td>
</tr>
<tr>
<td>60</td>
<td>51</td>
<td>930</td>
<td>1.33</td>
<td>1062</td>
<td>13.1</td>
<td>1.25</td>
</tr>
</tbody>
</table>
Site Characterization

- The conditions at this site consist of [top to bottom]:
  - Thin layer of loose, silty sandy fill [1 to 9 feet thick]
  - Loose to medium dense silty sand [5 to 25 feet thick]
  - Medium dense silty sand [0 to 35 feet thick]
  - Dense to very dense silty sand [thickness unknown, bottom of boreholes]
  - Groundwater located between 117 and 123 feet b.g.s.

- Shear wave velocity measurements were attempted at the site via the seismic cone penetrometer (SCPT), but were unsuccessful due to excessive background noise caused by traffic on the adjacent freeway.

- The probabilistic seismic hazard analysis (PHSA) assumed that the site classified as Site Class D (stiff soil profile) with an estimated average shear wave velocity of 270 m/s in the upper 100 feet.
Subsurface Profile

- M. Dense Silty Sand with Clayey Silt
- Dense to Very Dense Silty Fine Sand
- Loose Silty Sand Fill
- V. Stiff Lean to Fay Clay

Station: 4065 4066 4067 4068 4069 4070 4071 4072 4073 4074 4075 4076 4077 4078 4079 4080 4081 4082 4083 4084
Elevation: 940 950 960 970 980 990 1000
Seismic Hazard

- The site is not located in a mapped fault rupture zone; however due to the presence of several nearby, active faults the expected ground motion at this site is large.

- AREMA specifies 3 design earthquakes:
  - Level 1, 2 and 3 events represent spectral accelerations having average return periods of 100, 475 and 2475 years, respectively.

**Design Horizontal and Vertical Response Spectra**
Design and Performance Criteria (AREMA)

- **Level 1** – The embankment structure should remain intact with no permanent deformation (i.e. the seismic loads must remain within the elastic range of the stress-strain curve of the embankment);

- **Level 2** – The embankment structure should be repairable, with only minor permanent deformation; and

- **Level 3** – The embankment structure must not collapse after experiencing permanent deformations.
Preliminary Stability Analyses

- Comprehensive seismic design guidance for free standing lightweight concrete embankments has not been fully developed in the U.S.
- Rational numerical modeling approach applicable to the seismic design and evaluation of freestanding geofoam embankments.
- Principals from this approach applied in preliminary analyses.
- The geofoam is treated as a SDOF system (Horvath, 1996) and its fundamental period is calculated from Horvath (2004):

\[ T_0 = 2\pi \sqrt{\frac{\sigma'_v H}{(Eg)[4(H/B)^2 + \frac{12}{5}(1+\nu)]}} \]

- where: \( T_0 \) is the fundamental period, \( \sigma'_v \) is the vertical effective stress acting on the top of the geofoam from dead loads, \( H \) is the geofoam embankment height, \( E \) is the initial Young’s modulus of the geofoam, \( g \) is the gravitational constant, \( B \) is the width of the geofoam embankment and \( \nu \) is Poisson’s ratio.
- Preliminary stability analyses performed using limit equilibrium method with computer program SLOPE/W.
- Newmark pseudo-static sliding block displacement analyses used to provide preliminary estimate of deformation at each design spectra.
Preliminary Stability Analyses

AREMA Design Spectrum

- Level 1, $a_h = 0.31g$
- Level 2, $a_h = 0.93g$
- Level 3, $a_h = 1.60g$
Development of Strong Ground Motion

- More elaborate deformation analyses were also conducted that required horizontal and vertical acceleration time histories for the design basis events.
- These were developed as spectrum compatible time histories using spectral matching techniques from the computer program RSPMATCH (Abrahamson, 1998)
- Goal is to generate a set of realistic time histories satisfying seismological and geological conditions appropriate for the seismic source and site conditions at the candidate site.
- The main considerations for selecting time histories are:
  - appropriate earthquake magnitude,
  - faulting mechanism (e.g., strike-slip, vs. dip-slip, etc.)
  - tectonic regime,
  - source-to-site distance, and
  - geological structure.
- Note: The candidate records should be selected from earthquake events that have similar conditions, whenever possible.
Spectral Matching

Spectrally matched time histories for Level 3 horizontal response spectra.

Comparison of spectrally-matched time histories with Level 3 target spectrum
The spectrally-matched, strong motion records were deconvolved to a depth equal to the base of the 2D numerical model using the 1D equivalent linear procedures described by Mejia and Dawson (2006).

The deconvolution analysis was done with PROSHAKETM using linear-elastic soil properties for the foundation soils.

The deconvolution motion was convolved back to the surface to verify that the deconvolution-convolution analyses were capable of reproducing the original spectrally-matched time history at the ground surface.

Final Seismic Stability and Load/Deformation Analyses

- The computer programs QUAKE/W (Geostudio, 2007) and FLAC 2D (Itasca, 2005) were used to perform the seismic stability and load-deformation analyses of the embankment.

- The finite element option in QUAKE/W was used to model the embankment and foundation soil elastically, with no basal sliding interface, thus maximizing the seismic stresses induced in the embankment structure.
  - In QUAKE/W, the Direct Integration Method is used to compute the motion and predict excess pore-water pressures, if groundwater were present, resulting from inertial forces at user-defined time steps.

- The computer program FLAC was used to analyze the potential for yielding within the embankment, basal sliding, uplift and rocking of the embankment and to model the performance of the proposed ground improvement in the foundation soils.
Locations of History Points
QUAKE/W Equivalent Linear Elastic Analyses

UPRR Cotton Crossing
STA 8092+00

Description: AREMA Level 3 EQ
Date: 2/22/2011
Name: Equivalent Linear Dynamic
Kind: QUAKE/W
Method: Equivalent Linear Dynamic
Horz. EQ Record: CAPMED9.acc
Vert. EQ Record: CAPMEDV.acc
Elapsed Time: 4.26 sec
Selected Contour: X-Acceleration

MATERIALS

Name: Loose Silty Sand Fill Model: Linear Elastic Unit Weight: 115 psf Poisson’s Ratio: 0.25 Damping Ratio: 0.05 G Modulus: 67540 psf
Name: Dense to M Dense Silty Sand Model: Linear Elastic Unit Weight: 110 psf Poisson’s Ratio: 0.3 Damping Ratio: 0.06 G Modulus: 332450 psf
Name: Balsa/Strut/Backfill Model: Linear Elastic Unit Weight: 110 psf Poisson’s Ratio: 0.55 Damping Ratio: 0.05 G Modulus: 1114111 psf
Name: Class II Cellular Concrete Model: Linear Elastic Unit Weight: 25 psf Poisson’s Ratio: 0.22 Damping Ratio: 0.05 G Modulus: 386460 psf
Name: Precast Panel Model: Linear Elastic Unit Weight: 150 psf Poisson’s Ratio: 0.25 Damping Ratio: 0.05 G Modulus: 207840000 psf
Name: Loose Silty Sand Fill (improved) Model: Linear Elastic Unit Weight: 115 psf Poisson’s Ratio: 0.26 Damping Ratio: 0.98 G Modulus: 90000 psf
Name: Loose to M Dense Silty Sand (improved) Model: Linear Elastic Unit Weight: 115 psf Poisson’s Ratio: 0.25 Damping Ratio: 0.05 G Modulus: 1114111 psf
### Summary of Seismic Accelerations and Stresses at Level 2 Event

<table>
<thead>
<tr>
<th>History Point</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td>X-Acceleration (%g)</td>
<td>0.85</td>
<td>0.85</td>
<td>0.72</td>
<td>0.66</td>
<td>0.7</td>
<td>0.68</td>
<td>0.67</td>
</tr>
<tr>
<td>Y-Acceleration (%g)</td>
<td>0.96</td>
<td>0.32</td>
<td>0.30</td>
<td>0.28</td>
<td>0.28</td>
<td>0.56</td>
<td>0.55</td>
</tr>
<tr>
<td>XY-Shear Stress (psf)</td>
<td>245</td>
<td>159</td>
<td>948</td>
<td>1,198</td>
<td>1,147</td>
<td>2,995</td>
<td>4,817</td>
</tr>
<tr>
<td>X-Stress (psf)</td>
<td>498</td>
<td>243</td>
<td>162</td>
<td>492</td>
<td>790</td>
<td>5,991</td>
<td>6,785</td>
</tr>
<tr>
<td>Y-Stress (psf)</td>
<td>572</td>
<td>230</td>
<td>1,198</td>
<td>1,790</td>
<td>2,200</td>
<td>2,749</td>
<td>8,597</td>
</tr>
</tbody>
</table>

### Summary of Seismic Accelerations and Stresses at Level 3 Event

<table>
<thead>
<tr>
<th>History Point</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td>X-Acceleration (%g)</td>
<td>1.72</td>
<td>1.48</td>
<td>1.19</td>
<td>0.98</td>
<td>0.97</td>
<td>0.97</td>
<td>0.97</td>
</tr>
<tr>
<td>Y-Acceleration (%g)</td>
<td>1.50</td>
<td>0.50</td>
<td>0.47</td>
<td>0.44</td>
<td>0.44</td>
<td>0.92</td>
<td>0.90</td>
</tr>
<tr>
<td>XY-Shear Stress (psf)</td>
<td>495</td>
<td>273</td>
<td>1,607</td>
<td>1,984</td>
<td>1,789</td>
<td>4,590</td>
<td>7,315</td>
</tr>
<tr>
<td>X-Stress (psf)</td>
<td>693</td>
<td>275</td>
<td>196</td>
<td>816</td>
<td>1,113</td>
<td>9,296</td>
<td>10,568</td>
</tr>
<tr>
<td>Y-Stress (psf)</td>
<td>712</td>
<td>263</td>
<td>1,382</td>
<td>2,058</td>
<td>2,518</td>
<td>3,460</td>
<td>12,404</td>
</tr>
</tbody>
</table>
According to AREMA (2010), the maximum useable strain at the extreme concrete compression fiber is equal to 0.003 in/in at concrete crushing, which corresponds to the ultimate load or extreme event.

The maximum shear strength of unreinforced cellular concrete was assumed to be, $V_c = 0.75x2x\sqrt{f'c}$ per ACI 213R-87 (ACI, 1999).

Allowable compressive strength taken as 30% of the ultimate compressive strength ($0.3\times f'c$) based on AREMA Section 2.26.1 for bearing on a loading area.

Allowable Strain 0.003 in/in

The ultimate compressive stresses (Y-Pressure) of Class II and Class IV cellular concrete are 20,160 psf (140 psi) and 59,328 psf (412 psi), respectively.

Maximum shear stress for the Class II and Class IV cellular concrete are 2,556 and 4,384 psf, respectively.

All computed stresses and strains from QUAKE/W analyses were below allowable
FLAC Non-Linear Analyses

LEGEND
18-Feb-11 23:22
step 178003
Dynamic Time 1.5000E+01
-4.176E+00 <= 7.917E+01
-3.693E+01 <= 4.643E+01

User-defined Groups
- User DM0 Silty Sand
- User LIM Dense Silty Sand
- User Fill
- User LIM Dense Silty Sand
- User Fill Improved
- User Class II Cellular Con
- User Class M Cellular Con
- User Ballast

Grid plot

interface id's
- 0 2E 1

Net Applied Forces
- mvector = 3.515E+05
- D 1E 6

Steven Bartlet
University of Utah

FLAC (Version 5.00)
Comparison of QUAKE/W & FLAC Analyses

- The results of both models found to be very complimentary and similar to each other.
- Both models were able to convolve the design ground motions to the surface accurately.
- Both models indicate some deformation of the shear key at its interface with the soil.
- QUAKE/W model was found to be very useful in estimating conservative seismic accelerations and internal forces at various points throughout the embankment and soil structure.
- FLAC model was very useful in evaluating the permanent deformation of the embankment and its interface with the foundation.
Findings and Conclusions

• With the incorporation of the proposed ground improvement and light weight cellular concrete shear key, seismic global stability analyses indicate that the cellular concrete embankment will remain stable under AREMA Level 1 seismic loading (F.S. > 1.0)

• Permanent displacement of the highest embankment structure section is expected to range from 1 to 4 inches at the Level 2 earthquake, and from 4 to 7 inches at the Level 3 earthquake.

• Cellular concrete embankment will not yield under any of the AREMA Level 1, 2 and 3 earthquakes, and its response should remain elastic.

• Shear key is integral to limiting the basal sliding of the embankment structure and is an important design feature.
Actual Construction Underway
Questions?